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**13.0 GENERAL**

Piers must support the vertical loads from the superstructure, and also the horizontal loads not resisted by the abutments. They must also be capable of resisting forces they may receive directly such as wind loads, floating ice and debris, expanding ice, hydrokinetic pressures, and vehicle impact. Factors affecting the type of pier to use for a given structure include the type of service under the structure, skew angle of service under structure, and the aesthetic contribution of the structure to the surrounding area.

The connection between pier and superstructure is usually a fixed or expansion bearing which allows rotation in the longitudinal direction of the superstructure. The bottom of pier footings on land are to be a minimum of 4 feet (1.2 meters) below finished groundline unless the footings are founded on solid rock. For footings located in streambeds or floodplains, the minimum is 6 feet (1.8 meters) below stable streambed elevation when the top of footing is below the streambed, unless the footings are founded on solid rock. Where a footing is placed on a concrete seal, the bottom of footing may be 4 feet (1.2 meters) below streambed provided the bottom of seal is at least 8 feet (2.4 meters) below streambed.

Footing excavation adjacent to railroad tracks which falls within the critical zone shown on Standard 38.1 requires an approved shoring system. Erosion protection is required for all excavations.

Except for Pile Encased Piers (Stand. 13.3) all footing concrete (except seal concrete) for footings under water shall be poured in the dry. Successful underwater concreting requires special concrete mixes and additives and placement procedures, and the risk of error is high. A major concern in underwater concreting is that the water in which the concrete is placed will wash away cement and sand or mix with the concrete and increase the water to cement ratio. It was previously believed that if the lower end of the tremie is kept immersed in concrete during a placement that the new concrete flows under and is protected by previously placed concrete. However, tests performed at the University of California-Berkeley show that concrete exiting a tremie pipe may exhibit many different flow patterns exposing more concrete to water than expected. Laitance, which is a layer of soft, weak, water-laden mortar may also form within the pour. The most important concrete property that determines how concrete will flow after exiting a tremie pipe is shear resistance. Slump tests do not measure shear resistance.

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### 13.01 PIER TYPES

The types of piers most frequently used in Wisconsin generally can be classified in one of the following categories:

1. Multi-Column Frames
2. Pile Bents
3. Pile Bents with Concrete Encasement Wall
4. Solid Single Shaft

All types of piers with the exception of Pile Bents are used for both grade separation structures and water crossings with limitations and restrictions. Pile Bents are not used to support structures over roadways or railroads due to the severe damage they may receive from vehicle impact.

#### (1) Multi-Column Frames

A Multi-Columned Pier as shown in Standard 13.1 is the type of pier most commonly used for grade separation structures. A minimum of 3 columns is required to provide redundancy in case of vehicle impact. If the pier cap cantilevers over the outside columns, a square end treatment is preferred over a rounded end treatment.

Multi-columned piers are also used for stream crossings. They are especially applicable where a long pier is required because of an extremely wide bridge or because the stream is skewed at a large angle to the bridge.

Either continuous or isolated footings are used, the choice being based on whichever is the most economical. If a cofferdam is required, its cost is included in the determination of the most economical type. If the column spacing is greater than 25 feet (7.5 meters), isolated footings are usually more economical.

See Standard 38.1 for further details on piers supporting bridges over railways.

A modification of the Multi-Columned Pier on Standard 13.1 is produced by omitting the cap and placing a column under each girder. This detail has been used for steel girders with wide girder spacings and the need for additional cross bracing at the bearings must be investigated. This modified pier is no longer a frame and is actually a series of Single Column Piers.

#### (2) Pile Bents

Pile Bents with either 12" or 14" (310 or 360 mm) steel HP piles or 12" or 14" (305 or 356 mm) diameter reinforced cast-in-place piles with steel shells (Timber Piles are not preferred for Pile Bents) may be used for small to intermediate stream crossings with the following limitations and restrictions:

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- (1) Maximum distance from top of pier cap to stable streambed elevation is:
    - (a) 15 feet (4.5 m) for 12" (310 mm) piles or 12" (305 mm) dia. piles
    - (b) 20 feet (6 m) for 14" (360 mm) piles or 14" (356 mm) dia. piles
  - (2) The structure shall have freeboard for the 100 year flood if the quantity of the 100 year flood exceeds 5,000 cu. ft. (140 cubic meters) per second.
  - (3) The minimum pile spacing shall be 3 feet (1 meter).
  - (4) If water velocity ( $Q_{100}$ ) is greater than 7 feet (2.1 m) per second the quantity of the 100 year flood shall be less than 12,000 cu. ft. (340 cubic meters) per second.
  - (5) If the streambed consists of unstable material, the velocity of the 100 year flood shall not exceed 9 feet (2.75 m) per second.
  - (6) All bearings supporting the superstructure are fixed type bearings or equivalent.
  - (7) The structure is located within Area 3 as shown in the "Facilities Development Manual", Procedure 13-1-15, Figure 1 and the piles are not exposed to water with characteristics that are likely to cause corrosion.

Pile bents with the height limitations stated in (1) are capable of resisting forces from floating ice and debris, expanding ice, or any other lateral forces that may occur. For HP piles, the x-axis of the pile is placed parallel to the pier C/L. The minimum reinforcing steel in the cast-in-place piles is 6 #7 (#22) bars in 12" (305 mm) piles and 8 #7 (#22) bars in 14" (356 mm) piles. The piles are designed as columns fixed from rotation in the plane of the pier at the top and at some point below streambed.

The piles are designed for vertical load from dead and live load and a simultaneous lateral force from floating and expanding ice or other floating debris. The lateral force used is equivalent to an ice pressure of 250 psi (1.7 MPa) acting on an area equal to the pile width times 12" (300 mm) which equals the equivalent ice thickness.

Because of the minimum pile spacing the type of superstructure for pile bents is generally restricted to concrete poured in place slabs, 36 in. (915 mm) prestressed girders, steel girders with spans under 70 feet (21 meters), and 21 inch (535 mm) and under precast prestressed box girders. Larger girders can also be used when site conditions are favorable. The minimum size of the pile cap for pile bents is 3 feet (1 meter) wide by 3 feet (1 meter) deep for 12" (310 mm) piles with the piles embedded 1'-9" (535 mm). For 14" (360 mm) piles the minimum size is 3 feet (1 meter) wide by 3'-6" (1.1 meters) deep. The minimum area of steel in the bottom of the cap is 4 #6 (#19) bars for 12" (310 mm) piles and 5 #6 (#19) bars for 14" (360 mm) piles. The outside piles are battered 2"/foot (165 mm/meter) and the inside piles are driven vertically.

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(3) Pile Bents with Concrete Encasement Wall

When the criteria for the use of Pile Bents are not completely satisfied, the use of a pile bent with the piles embedded in a concrete wall may be a useable alternative. The concrete wall provides greater resistance to lateral forces than an open pile bent pier. Also the hydraulic characteristics of a wall are superior to open piles which results in a smoother flow and reduces the likelihood of streambed erosion at high water velocities. Pile encased piers should not be used for normal water depths greater than 10 feet (3 m) as this is near the maximum practical depth for setting formwork and placing the bar steel.

Floating debris is also less likely to become lodged against a wall than in-between the piles of an open pile bent. Debris lodging between piles is not a serious problem unless the direction of the flow changes at flood stage and becomes nonparallel to the pier.

Pile bents with Concrete Encasement Walls with either 10", 12" or 14" (250, 310 or 360 mm) steel HP piles or 10 3/4", 12" or 14" (273, 305 or 360 mm) cast-in-place piles with steel shells may be used for small to intermediate stream crossings when the criteria listed under "Pile Bents" are not satisfied except for Numbers (3) and (6). When oil field pipe is furnished as an alternate a minimum diameter of 9 5/8" (245 mm) is required.

The concrete wall shall be a minimum of 2'-6 (750 mm) thick. The top 3 feet (1 meter) of the wall is made wider if a larger bearing area is required. See Standard 13.3. The bottom of the wall is placed 2' to 4' (600 mm to 1200 mm) below stable streambed elevation, depending upon stream velocities and depth of freezing. The concrete in the wall may be poured under water.

The following criteria are used for the design of concrete encased pier bents, including the cap steel:

- (1) The minimum pile spacing shall be 3 feet (1 meter).
- (2) Use a minimum of 5 piles per pier.
- (3) Design to resist forces from floating ice and other lateral forces that may occur, along with vertical loads due to dead and live load.
- (4) Design ice load is 250 psi @ 12" (1.7 MPa @ 300 mm) ice thickness. (Expanding ice pressure).
- (5) Check the connection between the superstructure and the pier.
- (6) All bearings supporting the superstructure are fixed type bearings or equivalent.

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Refer to Appendix B for an example of construction techniques required to get solid concrete below the streambed.

(4) Solid Single Shaft

Solid Single Shaft piers are used for all types of crossings. The choice between using a Multi-Column Frame and a Solid Single Shaft pier is based on economics or aesthetics. For high level bridges a Solid Single Shaft pier is generally the most economical and attractive pier type available. By using various shapes and proportions, their appearance can be greatly altered and many varieties can be created.

The type of Solid Single Shaft pier used most frequently is the Hammerhead Pier shown on Standard 13.2. The massiveness of this type of pier provides a large lateral load capacity to resist the somewhat unpredictable forces from floating ice and debris and expanding ice. They are used on the major rivers adjacent to shipping channels without additional pier protection. When used adjacent to railroad tracks, crash walls are not required.

Solid Single Shaft Piers can also be shaped in the form of an unbalanced "T". This results in additional horizontal clearance to a railroad or roadway passing under the structure at a skew. The use of unbalanced "T" Single Shaft Piers is very limited.

If a cofferdam is required and the upper portion of a Single Shaft Pier extends over the cofferdam, an optional construction joint is provided 2 feet (600 mm) above normal water. Since the cofferdam sheet piling is removed by extracting vertically, any overhead obstruction prevents removal and this optional construction joint allows the contractor to remove sheet piling before proceeding with construction of the overhanging portions of the pier.

A hammerhead pier is not used when the junction between cap and shaft would be less than the cap depth above normal water. The reason for this criteria is aesthetics since hammerhead piers are not very attractive when the shaft exposure above water is not significant. An alternative would be a wall type Solid Single Shaft pier or a Multi-Column Frame. If a wall type pier is used, both the sides and ends may be sloped if desired, and either a round, square, or angled end treatment is acceptable.

(5) Aesthetics

Refer to Appendix A for suggested alternative pier shapes. These shapes are currently being studied so no standard details are shown. It is desirable to standardize alternate shapes for efficiency and economy of construction. Use of these alternate pier shapes for aesthetics should be approved by the Chief Structures Development Engineer so that standard details can be developed.

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13.02 LOCATION

Piers are to be located to provide navigational clearance requirements and to give a minimum interference to flood flow. In general, place the piers parallel with the direction of stream current at flood stage. Make adequate provision for drift and ice by increasing span length and vertical clearances, by selecting proper pier types, and by using debris deflectors. Special precautions against scour are required in unstable stream beds. Reference may be made to Chapter 8.0 - Hydraulics.

In the case of railroad and highway separation structures, the spacing and location of piers and abutments is usually controlled by the clearances required for the roadways or tracks below the structure as well as the necessity to avoid existing improvements such as storm drains and sewers where it is not feasible to remove them. Requirements for vertical and horizontal clearances are specified in Chapter 3.0-Design Criteria.

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### 13.1 LOADS ON PIERS

The following loads are considered in the design of piers.

(1) Live Loads

The live load is the AASHTO loading that produces the maximum vertical reaction at the pier. Impact and distribution are not included.

The truck and lane loadings are assumed to occupy a width of 10 feet (3 meters). The lane load is uniformly distributed over the 10 feet (3 meters). The truck load is applied as two equal concentrated loads at 6 foot (1.8 m) spacing in the center of the 10 feet (3 meter) width.

The governing load is placed in "design traffic lanes" having a width  $W = W_c/N$ , where  $W$  = width of design traffic lane,  $W_c$  = roadway width between curbs exclusive of median strip, and  $N$  = number of design traffic lanes. Refer to AASHTO bridge specifications. The governing load is placed anywhere within the individual design traffic lanes to produce maximum stresses. When a pier is loaded with three or more "design traffic lanes" the live load is reduced by a probability factor.

If fewer lane loads are used than what the roadway width can accommodate, keep the loads within their "design traffic lanes". "N" in the formula  $W = W_c/N$  is the number of lane loads being applied.

For girder type superstructures the loads are transmitted to the pier through the girders. Simple beam distribution is used. The skew of the structure is not considered when calculating these girder reactions. Use the actual girder spacing and lane widths to determine the girder reactions applied at their actual locations on the pier.

For slab type superstructures the loads are assumed to be transmitted directly to the pier without any transverse distribution. This same assumption is used even if the pier cap is not integral with the superstructure. A truck load is applied as concentrated loads and a lane load as a uniform load. The skew of the structure is considered when applying these loads to the cap. The lane width is divided by the cosine of the skew angle and the load is distributed over the new lane width along the pier centerline. Impact is included in the load if there is no pier cap.

(2) Longitudinal Force

AASHTO requires that provision be made for a longitudinal force equal to 5 percent of

the lane live load with concentrated load for moment and no impact. This force acts in all lanes carrying traffic in the same direction and acts 6 feet (1800 mm) above the floor and the longitudinal component is applied at the bearings. It is meant to simulate the forces caused by vehicles braking or accelerating. It is not possible to transfer the moment of the longitudinal component acting above the bearing to the footing on typical bridge structures.

When designing for LRFD, the longitudinal force is either 25 percent of the axle weights of the design truck or tandem or 5 percent of the design truck or tandem plus lane load.

(3) Wind Loads

The basic wind load is for 100 mph (160 Kph) wind speeds assumed to act horizontally from any direction. Wind loads are divided into three types: (1) wind on live load, (2) wind on superstructure, and (3) wind on substructure.

AASHTO specifications are used for the usual girder and slab bridges having maximum span lengths of 125 feet (38 meters). This limitation does not apply to LRFD.

A. Wind Load on Live Load

100 lbs/linear ft. (1460 N per linear meter), transverse

\* 40 lbs/linear ft. (585 N per linear meter), longitudinal

\* Both forces are applied simultaneously.

B. Wind Load on Structure

50 lbs/sq. ft. (2.4 kPa), transverse, but not less than 300 lbs/ft.

\* 12 lbs./sq. ft. (0.575 kPa), longitudinal

\* Both forces are applied simultaneously.

These forces are more conservative than the forces given for more precise wind directions but are used for convenience since the difference is small.

\* No longitudinal forces are applied in LRFD.

C. Wind Load on Substructure

The wind force applied to substructure units is specified as 40 lbs./sq. ft. (1.92 kPa) both longitudinally and transversely.

An overturning force as specified is also applied.

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(4) Thermal Forces

The change in length of the superstructure and pier cap due to temperature changes cause deflections in the pier columns. These deflections cause "thermal forces" in the columns.

In determining the thermal forces applied to each substructure unit the entire bridge superstructure is considered. There is a neutral point on the superstructure which does move due to temperature changes. This point is determined by examination or by trial and error. Using trial and error a neutral point is assumed and all horizontal forces are computed. The point is shifted until all the horizontal forces are in equilibrium. The horizontal forces computed are the forces produced at each substructure unit by deflections due to temperature.

The thermal force on an expansion pier is the minimum of:

1. Dead load reaction times maximum bearing coefficient of friction.
2. Force caused by deflection of pier for given temperature change. The friction coefficients of bearings are given in Chapter 27.0 Bearings.

The thermal force on a single fixed pier in a bridge is the resultant of the unbalanced forces acting on the substructure units. Maximum friction coefficients are assumed for expansion bearings on one side of the bridge and minimum coefficients on the other side to produce the greatest unbalanced force on the fixed pier.

The thermal changes in length of the superstructure are assumed as being along the longitudinal axis of the superstructure regardless of the substructure skew angle. This assumption is more valid for steel structures than concrete structures.

The force on a column due to a thermal change in length of the superstructure is:

$$F = \frac{3EIaTL}{h^3 \times 144}$$

where: E = Modulus of Elasticity of column, Ksi  
I = Moment of Inertia of column, In.<sup>4</sup>  
a = Coefficient of Thermal expansion of superstructure  
T = Temperature change of superstructure, Degrees  
L = Expansion length of superstructure, Feet  
h = Column height, Feet  
F = Force per column, Kips

$$\text{Metric: } F = \frac{3EI\alpha TL}{h^3}$$

where: E = Modulus of Elasticity of column, MPa  
 I = Moment of Inertia of column, m<sup>4</sup>  
       Moment of Inertia for round concrete columns ( $\pi D^4/64$ )  
 a = Coefficient of Thermal expansion of superstructure  
 T = Temperature change of superstructure, Degrees C  
 L = Expansion length of superstructure, Meters  
 h = Column height, Meters

F = Force per column, MN

The following values apply to Wisconsin bridges for computing temperature forces. Do not confuse temperature change with temperature range used for expansion joints.

	<u>Reinforced Concrete</u>	<u>Steel</u>
Temperature Change	45°F (25°C)	90°F (50°C)
Coefficient of Thermal Expansion	See Chapter 28	

Temperature forces on bridges with two or more fixed piers are based on the movement of the superstructure along its centerline. These forces are assumed to act normal and parallel to the longitudinal axis of the pier as resolved through the skew angle. The lateral restraint offered by the superstructure is usually ignored.

(5) Force of Stream Current, Floating Ice and Drift

The force of flowing water on piers is given by AASHTO as  $P = KV^2$

where: P = Pressure, pounds/sq. foot

V = Water velocity, feet/second

K = Shape constant; 0.7 for circular piers, 1.4 for square ends, and 0.5 for angle ends of 30° or less.

Use the water depth and velocity at flood stage with the force acting at one-half the water depth. Normally this force does not govern the pier design.

The exact force of floating ice on a pier is difficult to determine. AASHTO sets this force at between 100 psi and 400 psi (0.7 MPa and 2.8 MPa), depending on the ice thickness and temperature at the time of breakup. Tests at the St. Lawrence Waterway

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showed crushing strengths of 300, 693 and 811 psi (2.1, 4.8 and 5.6 MPa) for temperatures of 28, 14 and 2°F (-2, -10 and -17°C) respectively. The crushing strength of ice depends on the rate of load application and temperature.

The force of floating ice on piers is caused by two conditions. First is the force of floating ice colliding with the pier as it floats downstream. By Newton's third law of motion,  $\text{Force} = M(V_1 - V_2)$ ;

where:  $M$  = Mass of ice.

$V_1$  = Velocity of ice before impact.

$V_2$  = Velocity of ice after impact.

Second is the force that develops when ice jams against the pier. The ice acts as a dam and transfers hydrostatic pressure to the pier, as well as the force due to current drag under the ice and wind action against the ice.

Investigate each site for existing conditions. If no data is available, use the following data as minimum design criteria.

1. Ice pressure is 250 psi (1.7 MPa).
2. Minimum ice thickness is 12 inches (300 mm).
3. Height on pier where force acts is midway between high and low water elevations.
4. Pier width is the projection of the pier perpendicular to stream flow.

(6) Force Exerted by Expanding Ice Sheet

Expansion of an ice sheet, as the result of a temperature rise after a cold wave, can develop considerable force against abutting structures. This force can result if it is restrained between two adjacent bridge piers or between a bluff type shore and bridge pier. Its direction is therefore transverse to the direction of stream flow.

Force from ice sheets depends upon thickness, maximum rate of air-temperature rise, extent of restraint of ice, and extent of exposure to solar radiation. In the absence of more precise information, estimate an ice thickness and a force of 8.0 ksf.

It is not necessary to design all bridge piers for expanding ice. If one side of a pier is exposed to sunlight and the other side is in the shade along with the shore in the vicinity of the pier, consider the development of pressure from expanding ice. If the central part of the ice is exposed to the sun's radiation, consider the effect of solar energy which causes

the ice to expand.

Forces from floating ice and expanding ice do not act on a pier at the same time. Consider each force separately when applying these design loads.

(7) Buoyancy

The footings of piers under water are designed for the buoyant effect of the water.

Full hydrostatic pressure based on the water depth measured from the bottom of the footing is assumed to act on the bottom of the footing. The upward buoyant force equals the volume of concrete below the water surface times the unit weight of water. The effect of buoyancy on column design is usually ignored. Use high water elevation when analyzing the pier for over-turning. Use low water elevation to determine the maximum vertical load on the footing.

The submerged weight of the soil above the footing is used for calculating the vertical load on the footing. Typical values are as follows:

	lbs/ft <sup>3</sup>	(kN/m <sup>3</sup> )			
	<u>Sand</u>	<u>Sand &amp; Gravel</u>	<u>Silty Clay</u>	<u>Clay</u>	<u>Silt</u>
MIN. (Loose)	50 (7.85)	60 (9.43)	40 (6.28)	30 (4.71)	25 (3.93)
MAX. (Dense)	85 (13.55)	95 (14.92)	85 (13.35)	70 (11.00)	70 (11.00)

(8) Centrifugal Force

Centrifugal force is included in the design of piers for structures on horizontal curves. The centrifugal force is the percentage  $C=0.00117S^2D$  or  $C = 6.68S^2/R$  of each axle load, excluding impact, of one standard truck in each traffic lane,

where: S = design, speed, mph  
D = Degree of curve  
R = Radius of Curve, feet

In LRFD, the centrifugal force is taken as the product of the axle weights of the design truck or tandem and the factor  $C = 4/3 V^2/gR$ . Lane loads are not included. (LRFD increases the force over LFD by 4/3).

where: V = design speed, Ft./sec.  
g = gravitational acceleration, 32.2 Ft./sec.  
R = Radius of Curve, Ft.

The centrifugal force is assumed to act radially and horizontally 6 feet (1800 mm) above the roadway surface. The point 6 feet (1800 mm) above the roadway surface is measured from the centerline of roadway. The design speed may be determined from the Wisconsin "Highway Design Manual", Chapter 11. It is not

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necessary to consider the effect of superelevation when centrifugal force is used.  
The point of application of centrifugal force considers this.

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**13.2 OFFICE PRACTICE FOR LOAD APPLICATIONS**

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**(1) Loading Combinations**

Piers are designed for the loading combinations specified by AASHTO except:

- 1. Impact force is not included.
- 2. Group VII is not used.

For LRFD Strength I, III and V conditions are checked.

**(2) Expansion Piers**

Transverse forces applied to expansion piers include one-half the adjacent span lengths. Longitudinal forces except for temperature include one-half the adjacent span lengths provided these forces do not exceed the maximum friction force (dead load times maximum friction coefficient of bearings). The maximum friction force is used for the temperature force as it is assumed the superstructure moves due to temperature changes.

**(3) Fixed Piers**

Transverse forces applied to fixed piers include one-half the adjacent span lengths. Longitudinal forces except for temperature include the entire bridge length. The magnitude of these forces is reduced by the amount of force taken by expansion piers and abutments. If there are two or more fixed piers, proportion the total bridge length between them. When temperature forces are considered, it is assumed the expansion bearings do not take any longitudinal forces. In this case the fixed pier carries longitudinal forces for the bridge length plus the unbalanced friction force in the expansion bearings. Do not design fixed piers for longitudinal forces that are less than the forces based on one-half the adjacent span lengths.

**(4) Point of Application of Forces**

Longitudinal forces are transmitted from the superstructure to the pier through the bearings. The effect of load application above the roadway has a small effect in axial loads applied to the adjacent piers and does not affect the horizontal loads. Transverse forces are applied at the points specified by AASHTO as these forces are effective in increasing transverse moments.

Transverse and longitudinal superstructure forces are converted to transverse and longitudinal forces of the pier by using the sine and cosine functions of the skew angle of the pier.

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Wind loads are applied to both sides of the structure to get the maximum effect. Use the precise wind loading specified by AASHTO instead of the approximate wind loading if a significant cost saving results.

Temperature changes in the superstructure cause it to expand and contract along its longitudinal axis. These length changes induce forces in the substructure units which are applied normal and parallel to the pier centerline as resolved by the sine and cosine of the skew angle. This assumption is applied to most structures.

Theoretically the superstructure expands along its centerline and the substructure unit must follow. This means the forces transmitted to the substructure are based on the stiffnesses of the pier in both the transverse and longitudinal directions. Where there are two or more fixed piers, there is movement in the piers due to temperature changes. For skewed piers these forces become excessive as the stiffness along the pier centerline is large. Except in unusual cases, this approach is ignored and the forces are resolved by the sine-cosine functions.

#### Caution

If there are two or more fixed piers that are very rigid, transverse cracking may occur in the deck superstructure due to thermal contraction and shrinkage between the fixed piers. This has occurred on some bridges. Consider an expansion detail for this situation.

Longitudinal cracks have occurred on wide superstructures due to fixity to the pier cap. Consider expansion details for this situation or a longitudinal joint using a sealer other than compression seals.

On grades over 2 percent the superstructure tends to move downhill towards the abutment. The low end abutment should be designed as fixed and the expansion joint or joints placed on the uphill side or high end abutment.

### 13.3 PIER FRAME ANALYSIS

The dimensions of the pier frame are assumed for the first trial for the loads to be carried by the pier. The results of the first trial are used to design the individual components of the pier; cap, columns, and footings. If the original size assumptions are incorrect, the pier dimensions are adjusted and the pier analyzed again until a satisfactory design is made.

Pier frames in general use round columns. Pier caps may be designed to any length. However, if the length between the outer column of the pier cap exceeds 65 feet (20 meters), special design considerations may be necessary due to changes in length caused by temperature.

The minimum thickness of spread footings is 2 feet (600 mm) and 2'-6 (750 mm) for pile footings. Isolated footings are used under individual columns unless the clearance between footings is less than 4'-6 (1400 mm); then a continuous footing is used. The degree of restraint of the footings is assumed as 100% for footings on rock (less if footing is in uplift), 70% for footings on piles, and 50% for footings on soil.

The column spacing on pier frames is to be 25 feet (7.5 meters) maximum. Column height is determined by the roadway elevation and the location of the footing criteria. Columns are assumed rigidly connected to the pier cap where used.

A pier cap is required on continuous slab span structures to facilitate replacement of the slab during future rehabilitation.

If the girder spacing in a structure exceeds 12 feet (3.5 meters), consider placing a column under each girder and eliminating the pier cap. The diaphragm connections of the girders are used to transfer the lateral loads between tops of the columns. If this type of structure is used in water, do not eliminate the concrete pier cap.

The pier is analyzed as a frame bent by any of the available analysis procedures considering side sway of the frame due to loading. The gross concrete areas of the components are used to compute their moments of inertia for analysis purposes. The effect of the steel reinforcing is neglected.

Vertical loads are applied to the pier through the girders or the slab of a concrete bridge. The vertical loads are varied to produce the maximum moments and shears at various positions throughout the structure in combination with the horizontal forces. The effect of length changes in the cap due to temperature is also considered in computing maximum moments and shears. All these forces produce several loading conditions on the structure which must be separated to get the maximum effect at each point in the structure. The maximum moments, shears, and axial loads from the analysis routines are used to design the individual components of the pier.

### 13.4 PIER CAP DESIGN

The following steps are recommended for the design of a pier cap.

- (1) Draw the maximum moment and shear curves using the results of the analysis routine.
- (2) Use the assumed cap cross section to determine the bar steel required for positive and negative moments based on load factor design. Bars are placed straight in the cap. Determine bar cutoff points. If the pier cap is cantilevered over exterior columns, the top negative bar steel may be bent down at the ends to develop end anchorage.

The minimum cap dimension to be used is 3 feet (1000 mm) deep by 2'-6" (750 mm) wide except a 2'-6" (750 mm) deep section may be used for caps under slab structures. If a larger cap is needed, use 6 inch (150 mm) increments to increase the size. The cap width is 1 1/2" (40 mm) wider than the column on each side to facilitate construction forming.

On continuous slab structures, the moment and shear forces are proportional between the transverse slab section and the cap by the ratio of their moments of inertia. The effective slab width assumed for the transverse beam is the minimum of 1/2 the center to center column spacing or 8.0' (2.5 meters).

$$\text{Cap Moment} = (\text{Total Moment}) \times \frac{(\text{I of Cap})}{(\text{I of Cap}) + (\text{I of Slab})}$$

The concrete slab is to extend beyond the edge of pier cap as shown on Std.'s 18.1 and 18.2. If the cap is rounded, measure from the tangent line on the cap.

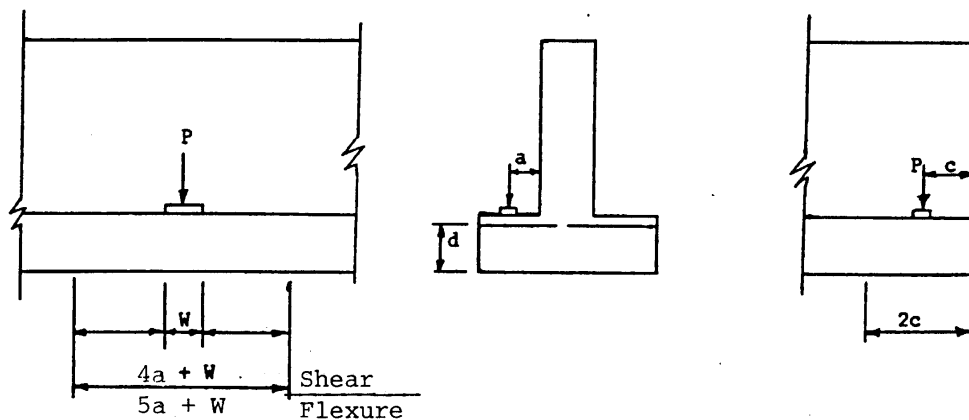
The pier cap is to extend a minimum of 2 feet (600 mm) beyond the centerline of bearing and centerline of girder intersection.

- (3) Determine the temperature steel required along the side faces of the cap ( $A_s = 0.11 A_g / f_y$  ( $A_g$  = Gross area of Section;  $f_y$  = yield strength of reinforcement). Longitudinal skin reinforcement needs to be equally distributed along the side faces of the member. The maximum bar spacing is 18". The area of skin reinforcement required = 0.012 (d-30) and need not exceed one-half of the required flexural tensile reinforcement.
- (4) Determine the need for stirrups to resist shearing forces and the size and spacing required. Do not place stirrups closer than 4 inch (100 mm) centers. Usually only double stirrups are used, but triple stirrups may be used to increase the spacing. If these do not work, increase the size of the cap. Stirrups are generally not placed over the columns. The first stirrup is placed one-half the spacing distance from the edge of the column into the span.

The cap to column connection is made by extending the bar steel in the column straight into the cap far enough to develop the bond strength. Stirrup details and bar details at the end of the cap are shown in Standard 13.1.

#### A. Design of Inverted T-Beams

An inverted T-beam is used as a bent cap beam in order to reduce the clearance requirements beneath the bent cap and to enhance the appearance of the bridge superstructure. The application of loads to the lower portion of the beam creates tensile forces not ordinarily encountered in T-beams. Reinforcement of the flange of the T presents special problems of shear, flexure and bar anchorage.



The sketches show the width of bracket that is effective to resist the concentrated load. For the interior portion of the beam:

$$\begin{aligned} \text{Effective Width} &= 4a + W \text{ (Shear)} \\ &5a + W \text{ (Flexure)} \end{aligned}$$

For loads near the end of the girder:

$$\text{Effective Width} = 2c \text{ provided } c \text{ is less than } 2.0a.$$

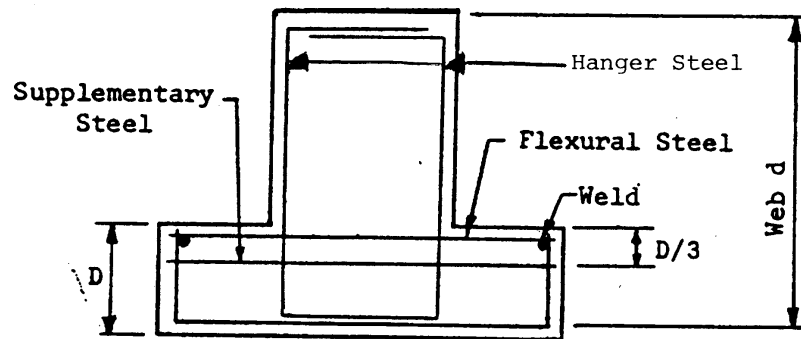
The depth of bracket  $d$  in inches required for shear friction is to be not less than:

$$d_{\min} = \frac{P}{0.8(4a + W) * 0.9}$$

where:  $P$  = ultimate load applied to bearing plate, Kips  
 $W$  &  $a$  = shown in sketch, inches

Within this effective width provide an area of reinforcement that extends into the flange to develop the shear friction force.

For flexural computations for the bracket reinforcement, the effective distance  $jd$  between the centroid of compression and the centroid of tension is taken as  $0.8d$ .



#### Flexural & Axial Steel

$$A_s = \frac{0.05(5a + W)}{f_y}$$

#### Hanger Steel (Service Limit State)

$$p = \frac{A_{hr} f_y * 0.5(W + 3a) * 0.9}{s}$$

where:

- $A_{hr}$  = Area of bar for Hanger Steel,  $\text{In.}^2$
- $P$  = Service Load, Kips
- $s$  = Spacing of hangers, In.
- $f_y$  = Yield strength of bars, ksi

The flexural steel has to be hooked to develop the tensile strength of the bar. In addition, Supplementary Steel with an area not less than half the Flexural Steel is placed as shown in the sketch. Nominal flexural steel reinforcing is to be provided between points of concentrated loads.

At every concentrated load applied to the flange of the inverted T-beam, provide stirrups that act as hangers that have a capacity greater than the applied service load. Place the stirrups within  $(W + 3a)$ . The hangers must be closed across the bottom to develop the stirrup bar forces.

The design of the beam to resist flexure along its horizontal axis is the same as any rectangular concrete beam. Stirrups along this axis are designed to resist all ultimate shears not resisted by the concrete.

### 13.5 COLUMN DESIGN

Use an accepted analysis procedure to determine the axial load and longitudinal and transverse moments acting on the column. These forces are found at the top and bottom of the column. Apply the designated load factors for each design group condition. Choose the larger loading cases for designing the column.

Columns that are part of a pier frame have transverse moments induced by frame action from vertical loads, wind loads on the superstructure and substructure, wind loads on live load, thermal forces, and centrifugal forces. Longitudinal moments are produced by the above forces plus the longitudinal live load force. These forces are resolved through the skew angle of the pier to act transversely and longitudinally to the pier frame. Longitudinal forces are divided equally among the columns.

| Wisconsin uses tied columns following AASHTO Specifications. The minimum column size used is 2'-6 (750 mm) in diameter. The minimum bar steel area is by specifications.

The computed column moments are magnified for slenderness effects as defined by AASHTO Specifications.

The computed moments are multiplied by the magnification factors and then the column is designed for the computed loads following the specifications. The design strength under combined flexure and axial load is based on stress and strain compatibility. A computer program is recommended for solution due to the complexity of the problem.

Large single column pier shafts such as "Hammerhead Piers" are designed as a wall. The minimum steel allowed is shown on the Standard.

On large river crossings it may be necessary to protect the piers from damage. The noses may be protected against damage by using steel with anchors embedded in the concrete. The column to cap connection is designed as a rigid joint considering axial and bending stresses. Column steel is run through the joint into the cap to develop the compressive stresses or the tensile steel stresses at the joint.

In general the column to footing connection is designed as a rigid joint. However, in cases of high transverse moments, it may be desirable to pin the connection in the transverse direction. If this is done, the bar steel is placed at the neutral axis of the column in the transverse direction as it passes through the joint. In all cases the column to footing connection is designed as a rigid joint in the longitudinal direction. The bar steel from the column is usually terminated at the top of the footing. Dowel bars are used to transfer the steel stress from the footing to the column.

## (1) Tapered Columns of Concrete and Timber

Design these columns using the existing column formulas, taking the cross sectional area at the small end, but  $d$ , the dimension used in  $L/d$ , is taken as follows:

1. For round columns or rectangular columns tapered in both directions, use  
 $d = d_B$
2. For rectangular columns tapered in the plane of bending only, use  
 $d = (d_A).2x(d_B).8$
3. For rectangular columns tapered perpendicular to the plane of bending, use  
 $d = (d_A).7x(d_B).3$

Where:  $d_A$  = dimension at the small end

$d_B$  = dimension at the large end

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### 13.6 FOOTING DESIGN

#### (1) Isolated Spread Footings

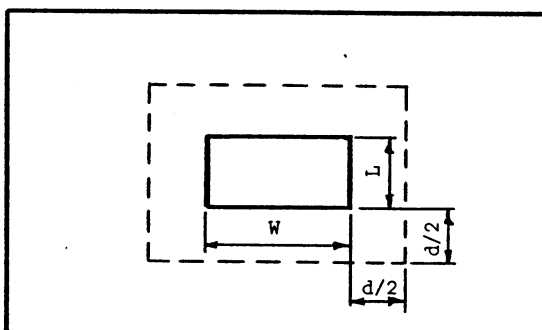
Spread footings are designed according to AASHTO Specifications. The footing is proportioned so when it is loaded with the Group Loadings, the basic allowable soil stress with the percent increase is not exceeded. The following criteria is used to design footings.

- A. Minimum depth of spread footings is 2 feet (600 mm). Depth is generally determined from shear strength requirements. Shear reinforcement is not used.
- B. A maximum of 25% of the area of the footing is allowed in uplift. When part of a footing is in uplift its section properties for analysis are based only on the portion of the footing in compression. When determining the percent of a footing in uplift, apply the actual load without any reductions for overload. For Groups 2-9 the basic allowable soil stress is increased but the allowable percent of area in uplift is 25. No uplift is allowed for Group 1 loading.
- C. Soil weight on footings is based only on the soil directly above the footing.
- D. The minimum depth for frost protection from top of ground to bottom of footing is 4 feet (1200 mm).
- E. Spread footings on seals are designed by either method as follows:
  - a. The footing is proportioned so the pressure between the bottom of the footing and the top of the seal does not exceed the allowable soil stress and not more than 25% of the footing area is in uplift.
  - b. The seal is proportioned so that pressure at the bottom of the seal does not exceed the allowable soil stress and the area in uplift between the footing and seal does not exceed 25%.
- F. The reinforcing steel in spread footings is determined from the requirements for bending as stated in AASHTO. The moment used is determined from the volume of the pressure diagram under the footing which acts outside of the section being considered. The weight of the footing and soil above the footing is used to reduce the bending moment.
- G. The negative moment which results from 25 percent or less of the footing area being in uplift is ignored. No negative reinforcing steel is used in spread footings.

H. Shear strength is determined by two methods.

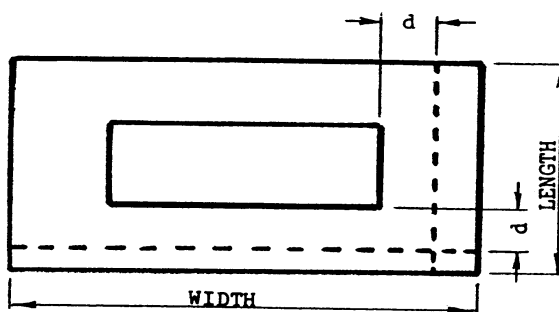
a. Two-way action

The volume of the pressure diagram on the area of the footing outside lines at a distance  $d/2$  from the face of column. The shear stress is determined by applying the forces from the volume of the pressure diagram over the length  $2(L + d + W + d)$  for rectangular columns and  $3.14 (2R + d)$  for round columns where  $R$  is the radius of the column and  $d$  the effective footing depth.



b. One-way Action

The volume of the pressure diagram on the area enclosed by the footing edges and a line at distance " $d$ " from the face of the column. The shear stress is determined by applying the forces from the volume of the pressure diagram over the entire footing width or length.



The weight of the footing and soil above the areas is used to reduce the shear force.

- I. The bottom layer of reinforcing steel is placed 3 inches (75 mm) clear from the bottom of the footing.
- J. If adjacent edges of isolated footings are closer than 4'-6 (1400 mm), use a continuous footing.

(2) Isolated Pile Footings

Pile footings are designed according to AASHTO Specifications. The footing is proportioned so when it is loaded with the Group Loadings the basic allowable pile stress, with the percent increase, is not exceeded. The following criteria is used to design the footings.

- A. Minimum depth of pile footing is 2'-6 (750 mm). The minimum pile embedment is 6" (150 mm).
- B. Pile footings in uplift are usually designed by method (a) stated below. However, method (b) may be used if the designer feels there is a substantial reduction in cost.
  - a. Over one-half of the piles in the footings must be in compression. The section properties used by analysis are based only on the piles in compression. When analyzing the footing, apply the actual load and use the allowable stress increases as stated in AASHTO. All piles must be in compression for Group 1 loading.
  - b. Piles may be designed for upward forces provided an anchorage device, sufficient to transfer the load, is provided at the top of the pile. Provide reinforcing steel to resist the tension stresses at the top of footing.
- C. Same as Spread Footing.
- D. Same as Spread Footing.
- E. The minimum number of piles per footing is four.
- F. Pile footings on seals are analyzed above the seal. The only effect of the seal is to reduce the allowable pile stress above the seal by the portion of the seal weight carried by each pile.

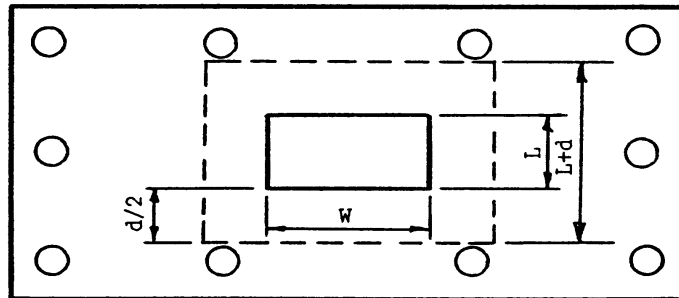
If no seal is required but a cofferdam is, design the piles to use the minimum required batter. This reduces the size of the cofferdam necessary to clear the battered piles as all piles extend above water to the pile driver during driving.
- G. The reinforcing steel in pile footings is determined from the requirements for bending as stated in AASHTO. The moment and shear used is determined

from the force of the piles which act outside of the section being considered. The weight of the footing and soil above the footing is used to reduce the bending moment and shear force.

H. Shear strength is determined by two methods

a. Two-way Action

The summation of the pile forces is on the area outside lines at a distance  $d/2$  from the face of the column (" $d$ " = the effective footing depth). The shear stress is determined by applying the force over the length  $2(L + d + W + d)$  for rectangular columns and  $3.14(2R + d)$  for round columns.



If the center of a pile falls on a line, 1/2 of the pile force is assumed to act on each side of the line.

b. One-way Action

The summation of the pile forces is within the area enclosed by the footing edges and a line at distance " $d$ " from the face of the column. The shear stress is determined by applying the force over the entire footing width or length.

The weight of the footing and soil above the areas is used to reduce the diagonal tension force.

I. The bottom layer of reinforcing steel is placed directly on top of the piles.

(3) Continuous Footings

Continuous footings are used in pier frames of two or more columns when the use of isolated footings would result in a distance of less than 4'-6" (1400 mm) between edges of adjacent footings. They are designed for the moments and shears produced by the frame action of the pier and the soil pressure under the footing.

The soil pressure or pile load under the footing is assumed to be uniform. The soil pressures or pile loads are generally computed only from the vertical column loads and soil and footing dead load. The moments at the base of the column are ignored for soil or pile loads.

To prevent unequal settlement, proportion the continuous footing so the soil pressures or pile loads are constant for Group 1 loading. The footing is kept relatively stiff between columns to prevent upward footing deflections which cause high soil or pile loads under the columns.

| (4) Seals and Cofferdams

A seal is the mat of unreinforced concrete poured under water inside the sheet piling of a cofferdam. It is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. Dewatering the cofferdam allows cutting of piles, placement of reinforcing steel and pouring of the footing in an air environment.

Seals are required for all piers founded on spread or pile footings that are too far below normal water to pour the footing in the dry. A seal is required to allow pouring the footing concrete in the dry to insure properly consolidated concrete. This is especially important for tall piers.

The hydrostatic pressure under the seal is resisted by the weight of the seal, the friction between the seal perimeter and walls of the cofferdam, and friction between seal and piles for pile footings. The friction values to use for seal design are considered working stresses so a factor of safety is not required. To compute the capacity of piles in uplift see Chapter 11 on Piling.

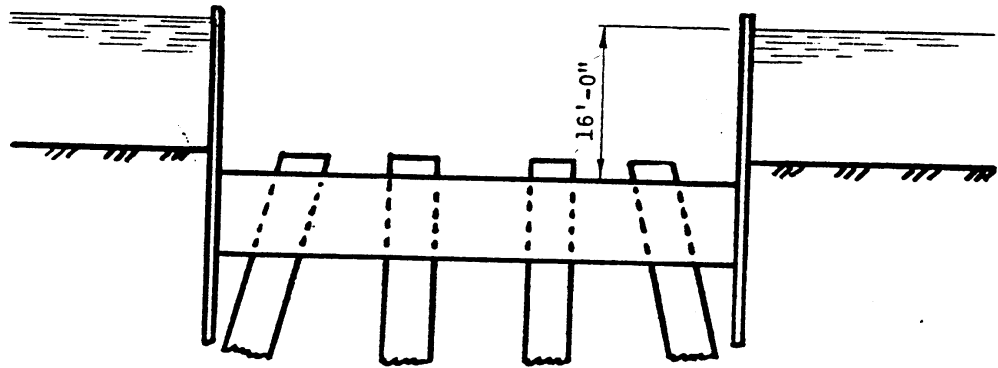
Bond on Piles	-	10 psi (70 kN/m <sup>2</sup> )
Bond on Sheet Piling	-	2 psi (14 kN/m <sup>2</sup> ) on (seal depth minus 2 feet (600 mm) x perimeter.

Lateral forces from stream flow pressure are resisted by the penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. The cofferdam design is the responsibility of the contractor. When seals for spread footings are founded on rock, the weight of the seal is used to counter balance the lateral streamflow pressure.

The downstream side of the cofferdam should be keyed into rock deep enough or backfilled enough to resist the lateral stream flow pressure. To provide a factor of safety the weight of the cofferdam (sheet piling and bracing) is ignored in the analysis. The design stream flow velocity is based on the flow at the site at the time of construction but need not exceed 75% of the 100 year velocity. Use AASHTO specifications to calculate the force. If stream velocity higher than design occurs, the contractor can add water to the cofferdam to improve stability. The extra weight of the water above the seal adds additional overturning resistance and also increases lateral resistance from additional friction forces at the bottom of the seal.

A rule of thumb for seal thickness is  $0.40 H$  for spread footings and  $0.25 H$  for pile footings where  $H$  is the depth of water from bottom of seal to top of normal water. The minimum seal size is 1'-6 (500 mm) larger than the footing size on all sides.

Example: Determine seal thickness for a 9' x 12' footing with 12 - 12" diameter piles.  
Uplift capacity of one pile = 10 Kips. Water depth to top of seal is 16'.  
Assume 12' x 15' x 3' seal



Uplift force of water - $12 \times 15 \times 19 \times .0624$	= 214 kips up
Weight of seal course - $12 \times 15 \times 3 \times .15$	= 81 kips down
Friction of sheet piling $2 \times (12+15) \times 1 \times 144 \times .002$	= 16 kips down
Friction on 12 inch diameter pile $p \times 12 \times 36 \times .010$	= 13.6 kips
Maximum uplift/pile	= 10 kips
Total available force from piles = $12 \times 10$	= 120 kips down
Summation of downward forces	= 217 kips

$217 > 214$  OK

USE 3'-0" THICK SEAL

| A cofferdam is required when the depth of water from the bottom of the seal to the top of  
| the water is 15 feet or greater. Cofferdams may be required when this depth is less than  
| 15 feet for certain soil conditions and stream flows. The designer should consult with  
| geotechnical personnel for these cases. A bid item for Cofferdam should be shown if the  
| possibility of one exists.

## (5) Isolated Spread Footing Sample Problem

NOTE: This sample problem is shown using english units for computations since this is still familiar to most designers.

Problem: Design a spread footing for a group 2 loading condition.

Axial load is 400 kips, transverse moment is 500 ft. kips, and longitudinal moment is 800 ft. kips. Allowable soil pressure equals 6 kips/sq. ft. Column size is 2' x 2'.  $f_c' = 3500$  psi,  $f_y = 60000$  psi. A 25% soil pressure overstress is allowed for group 2 loading.

## A. Determine Footing Size

Estimate a footing size.

Assume DL of footing and earth = 50 kips

From  $P/A = 6.0 \times 1.25 = 7.5$  ksf

(Group 2 Allowable soil pressure)

$A = 450./7.5 = 60$  sq. ft.

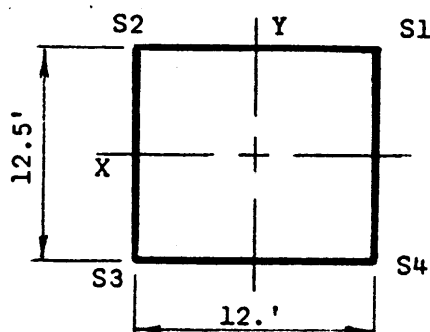
Because of the high moments assume the soil stress in one corner of the footing is near zero. Therefore the average stress on the bottom of the footing is half of 7.5 and the approximate required area is 120 sq. ft.

Try a 12' x 12.5' x 2' footing:

Concrete DL =  $12 \times 12.5 \times 2 \times .150 = 45.0$  Kips

Earth DL =  $(150-4) \times 2 \times .120 = 35.0$  Kips

Total DL = 80.0 Kips



$$A = 12. \times 12.5 = 150 \text{ sq. ft.}$$

$$S = 1/6 bh^2$$

$$S_T = 12.5/6 \times 12^2 = 300.$$

$$S_L = 12/6 \times 12.5^2 = 312.$$

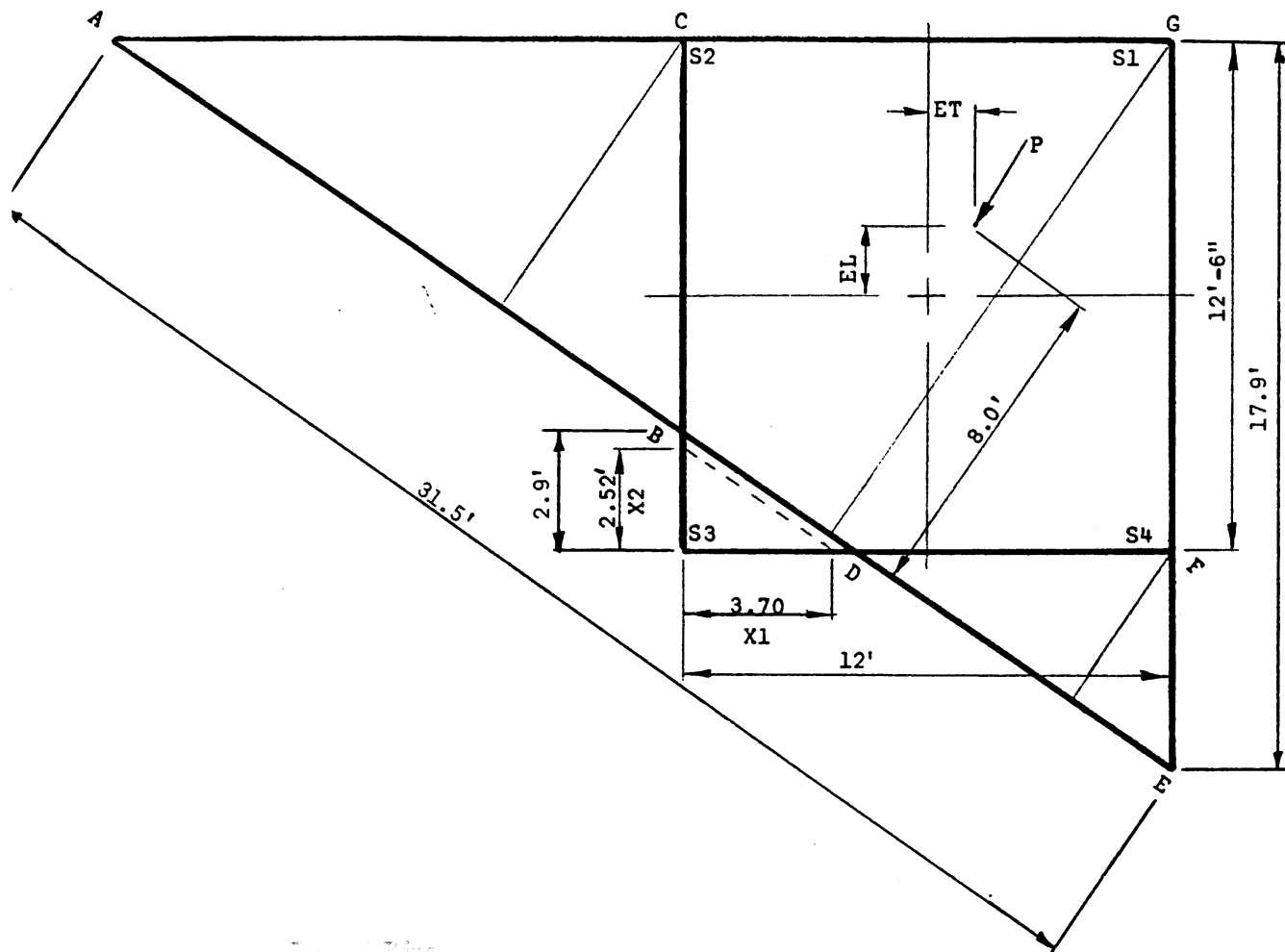
$$S1 = \frac{P}{A} + \frac{MT}{S} + \frac{ML}{S} = \frac{480}{150} + \frac{500}{300} + \frac{800}{312} = 7.43 \text{ Ksf}$$

$$S2 = \frac{P}{A} - \frac{MT}{S} + \frac{ML}{S} = 4.09 \text{ Ksf}$$

$$S3 = \frac{P}{A} - \frac{MT}{S} - \frac{ML}{S} = -1.03 \text{ Ksf}$$

$$S4 = \frac{P}{A} + \frac{MT}{S} - \frac{ML}{S} = 2.31 \text{ Ksf}$$

Since S3 is in tension, reanalyze the footing using only the portion of the footing in compression.



$P = 480$  Kips  
 $MT = 500$  Ft. Kips  
 $ML = 800$  Ft. Kips  
 $ET = 500/480 = 1.041$  Ft.  
 $EL = 800/480 = 1.668$  Ft.

Find points of zero stress

$$\begin{aligned}
 X1 &= 1.03 \times 12/3.34 = 3.70 \text{ ft.} \\
 X2 &= 1.03 \times 12.5/5.12 = 2.52 \text{ ft.}
 \end{aligned}$$

The actual line of zero stress is parallel to the line passing through the points of zero stress computed but closer to the point of maximum stress.

Select a trial zero axis 3 inches closer to the point of maximum stress. The footing is drawn to scale and the triangles ABC and DEF are drawn. The moment of inertia of BCGFD is computed about axis AE using  $I = bh^3/12$  for the large triangle minus the I's of the two smaller triangles. Distances b and h are determined by scaling.

$$I = 31.5 \times 14.75^3/12 - 17 \times 8^3/12 - 9.5 \times 4.4^3/12 = 7618 \text{ ft.}^4$$

$$\text{Moment about the line of zero stress is } P \times 8' = 480 \times 8 = 3840 \text{ ft. kips}$$

$$S1 = M/S = 3840/(7618./14.75) = 7.43 \text{ Ksf.}$$

$$S2 = 14/26 \times 7.43 = 4.00$$

$$S4 = 5.3/17.9 \times 7.43 = 2.20$$

Check the volume of the pressure diagram to see if it is equal to P. If the discrepancy is large, assume a new location of the line of zero stress and repeat the procedure.

$$\text{Volume of AEG} = 1/3 Ah = 1/3 \times 1/2 \times 31.5 \times 14.75 \times 7.43 = 574. \text{ Kips}$$

Volume of two small triangles:

$$ABC = 1/3 \times 1/2 \times 17 \times 8 \times 4.00 = 90.6 \text{ Kips}$$

$$DEF = 1/3 \times 1/2 \times 9.5 \times 4.4 \times 2.2 = 15.3 \text{ Kips}$$

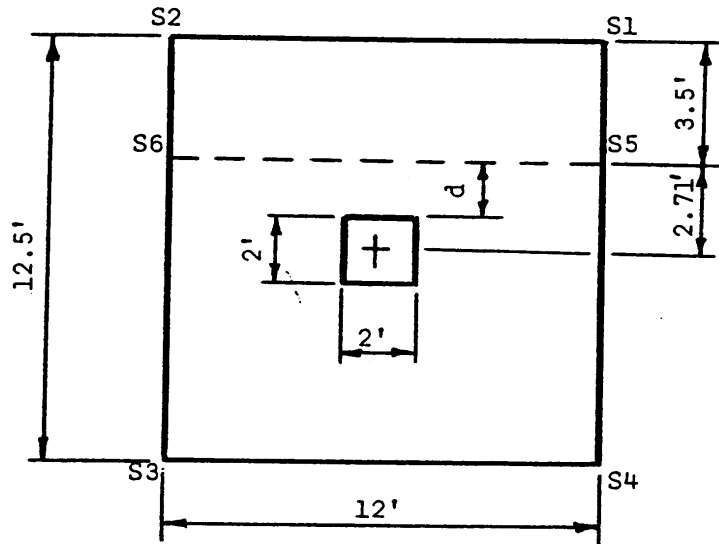
$$\text{Total Volume} = 574 - 91 - 15 = 468 \text{ Kips}$$

Since 468 is less than 480, the assumed portion of the line of zero stress is too close to the point of maximum stress. However, the stresses obtained are close enough.

If the corner stresses exceed the allowable value or the pressure volume is less than the axial load, it may be necessary to increase the footing size to get a satisfactory design.

## B. Check Shear Strength

Both one-way action and two-way action are checked. For group 2 loading the load factor is 1.3 for all loads. S1 and S2 therefore increase by a factor of 1.3.



$$S1 = 1.3(7.43) = 9.66 \text{ kips/sq. ft.}$$

$$S2 = 1.3(4.00) = 5.20$$

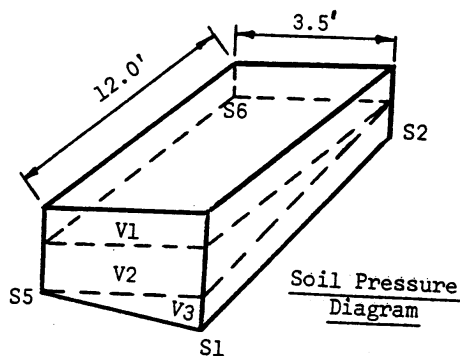
$$d = 24 - 3.5 = 20.5" = 1.71'$$

By proportioning:

$$S5 = 9.66(17.9 - 3.5)/17.9 = 7.77$$

$$S6 = 5.20(9.6 - 3.5)/9.6 = 3.30$$

Compute the volume of the soil pressure diagram that has the four corner stresses of S1, S2, S6 and S5.



$$V1 = S6(12)(3.5) = 138.6$$

$$V2 = (S5 - S6)(12)(3.5)(.5) = 93.9$$

$$V3 = (S1 - S5 + S2 - S6)(12)(3.5)/4 = 39.8$$

The dead load of the footing and earth must be deducted since they are included in S1, S2, S5 and S6.

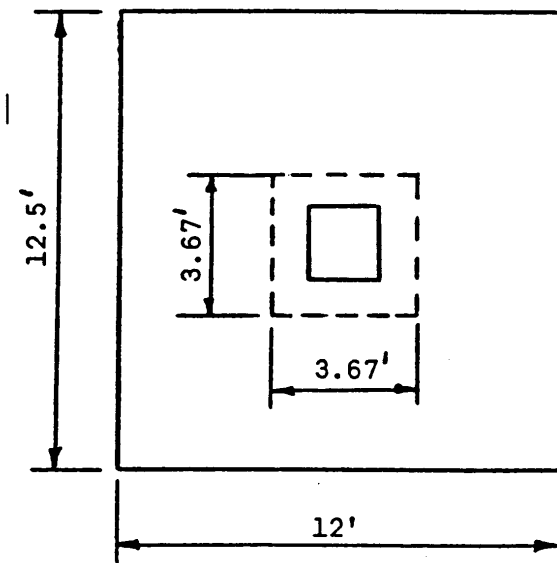
$$\text{Total Shear } V_u = V_1 + V_2 + V_3 - 3.5(12)(.300 + .240) = 272.3 - 22.6 = 249.7 \text{ kips}$$

$$\text{Actual Shear Stress } v_u = V_u / \phi b_w d$$

$$v_u = 249.7 \times 1000 / (.85 \times 144 \times 20.5) = 99 \text{ psi}$$

$$\text{Allowable Shear Stress } 2\sqrt{f_c'} = 2\sqrt{3500} = 118 \text{ psi}$$

Two-way action or peripheral shear must also be checked although it seldom governs. It is only necessary to consider the vertical reaction from the column.



$$P = 1.3(400) = 520. \text{ kips}$$

$$V_u = 520(12. \times 12.5 - 3.67^2) / (12 \times 12.5)$$

$$\text{Total Shear } V_u = 520(136.5) / 150 = 473.2$$

$$v_u = V_u / \phi b_o d$$

$$v_u = 473.2 / (.85 \times 4 \times 12 \times 3.67 \times 20)$$

$$\text{Actual Shear Stress } v_u = 473.2 \times 1000 / 2994.7 = 158. \text{ psi}$$

$$\text{Allowable Shear Stress} = 4\sqrt{f_c'} = 236. \text{ psi}$$

Use 2'-0 for footing depth

## C. Design Longitudinal Steel

Determine moment at face of column

$$S1 = 9.66 \text{ kips/sq. ft.}$$

$$S2 = 5.20$$

By proportioning:

$$S7 = 9.66(17.9 - 5.25)/17.9 = 6.83$$

$$S8 = 5.20(9.6 - 5.25)/9.6 = 2.36$$

Compute volume of soil pressure diagram of S1, S2, S8 and S7.

$$R1 = S8(5.25)(12.00) = 148.7$$

$$R2 = (S7 - S8)(12)(5.25)/2 = 140.8$$

$$R3 = (S1 - S7 + S2 - S8)(12)(5.25)/4 = 89.3$$

Compute moment of pressure diagram about line S7 to S8.

$$M = (R1 + R2)5.25/2 + R3(.667)(5.25)$$

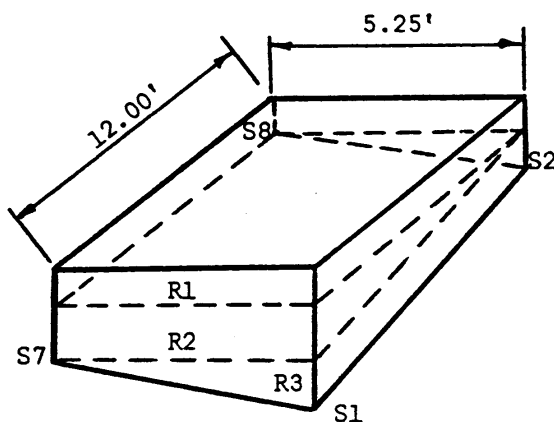
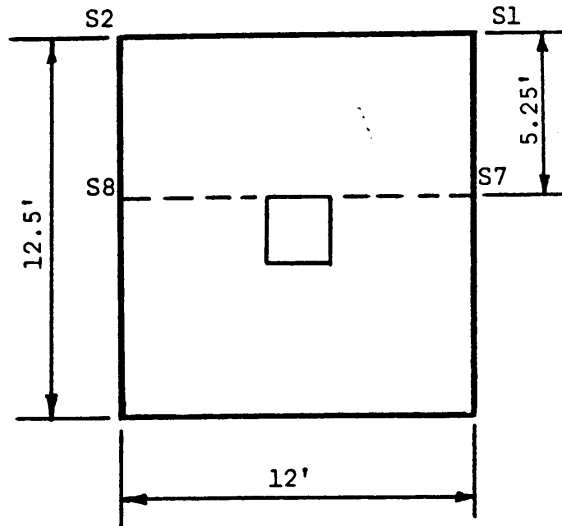
$$M = 759.9 + 312.7 = 1072.6 \text{ ft. kips}$$

Deduct moment due to footing and soil weight.

$$W = 12(5.25)(.3 + .24) = 34 \text{ kips}$$

Total moment:

$$M_u = 1072.6 - 34(1.3)(5.25)/2 = 956.6 \text{ ft. kips}$$

Soil Pressure Diagram

Effective Footing Width:

From AASHTO article 1.4.6(F) the band width  
=  $24 + 2(21) + .5(144 - 66) = 105$  inches

Compute steel area required.

$$R_u = M_u / \phi b d^2$$
$$R_u = 12(956.6) / (.9 \times 105 \times 20.5^2) = .289$$

From design tables for  $f_y = 60000$ . psi and  $f_c' = 3500$  psi

$$\text{RHO} = .0051$$

$$\text{Area of steel} = .0051(105)(20.5) = 10.98 \text{ sq. in.}$$

The required area and perimeter of steel per foot in the band width =  
 $10.98/8.75 = 1.25$  sq. in. per foot.

Use #9 @ 9" within band and #9 @ 1'-6 outside of band

The transverse steel is designed in a similar manner. The shear strength in the transverse direction does not govern.

C. Footing Dimensions

Footing size =  $12.5' \times 12' \times 2'$

Bar Steel - Use #9 @ 9 inches within band

Use #9 @ 1'-6 outside of band.

## (6) Isolated Pile Footing Sample Problem

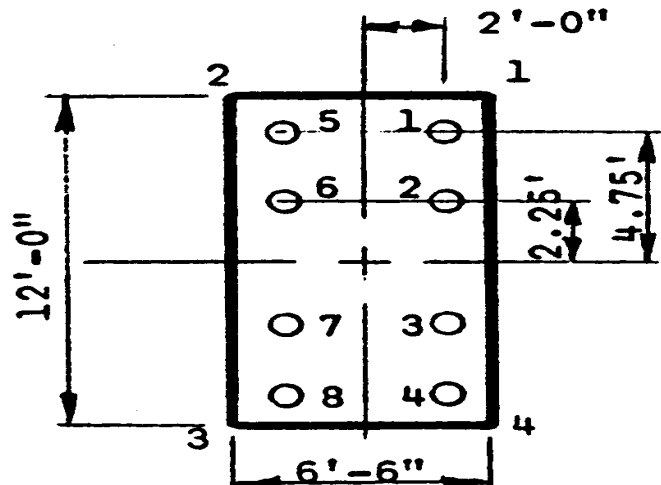
Note: English Units are used.

Problem: Design a pile footing for a group 2 loading condition.

Axial load is 400 kips, transverse moment is 500 ft. kips and longitudinal moment is 800 ft. kips. Allowable single pile capacity = 110 kips (100% friction pile). Column size is 2' x 2'. Assume piles offer no resistance to uplift forces. A 25% pile overstress is allowed for group 2 loading.

## A. Determine Footing Size and Piles Required

The number of piles is first estimated. Assume DL of footing and earth = 50. kips and average pile force = 60 kips. From  $P/A = 450/60 = 8$  piles.



Try a 6'-6" x 12'-0" x 3' footing

$$\begin{aligned} \text{Concrete DL} &= 12 \times 6.5 \times 3 \times .15 = 35.1 \\ \text{Earth DL} &= (78-4) \times 1 \times .12 = 8.9 \\ \text{Total DL} &= 44.0 \end{aligned}$$

Section Properties

$$S_T = 8x2^2 / 2 = 16 \text{ ft.}^3$$

$$S_L = (4x4.75^2 + 4x2.25^2) / 4.75 = 23.3 \text{ ft.}^3$$

$$N = 8$$

Pile loads are:

$$FL = \frac{P}{N} + \frac{M_T}{S_T} + \frac{M_L}{S_L}$$

$$F1 = 444./8. + 500/16 + 800/23.3 = 121.2 \text{ Kips}$$

$$F8 = 55.5 - 31.3 - 34.4 = -10.2 \text{ Kips}$$

$$F5 = 55.5 - 31.3 + 34.4 = 58.6 \text{ Kips}$$

$$F4 = 55.5 + 31.3 - 34.4 = 52.4 \text{ Kips}$$

Since F 8 is negative, reanalyze the footing using only the piles in compression. Determine the line of zero stress based on the previously computed pile loads:

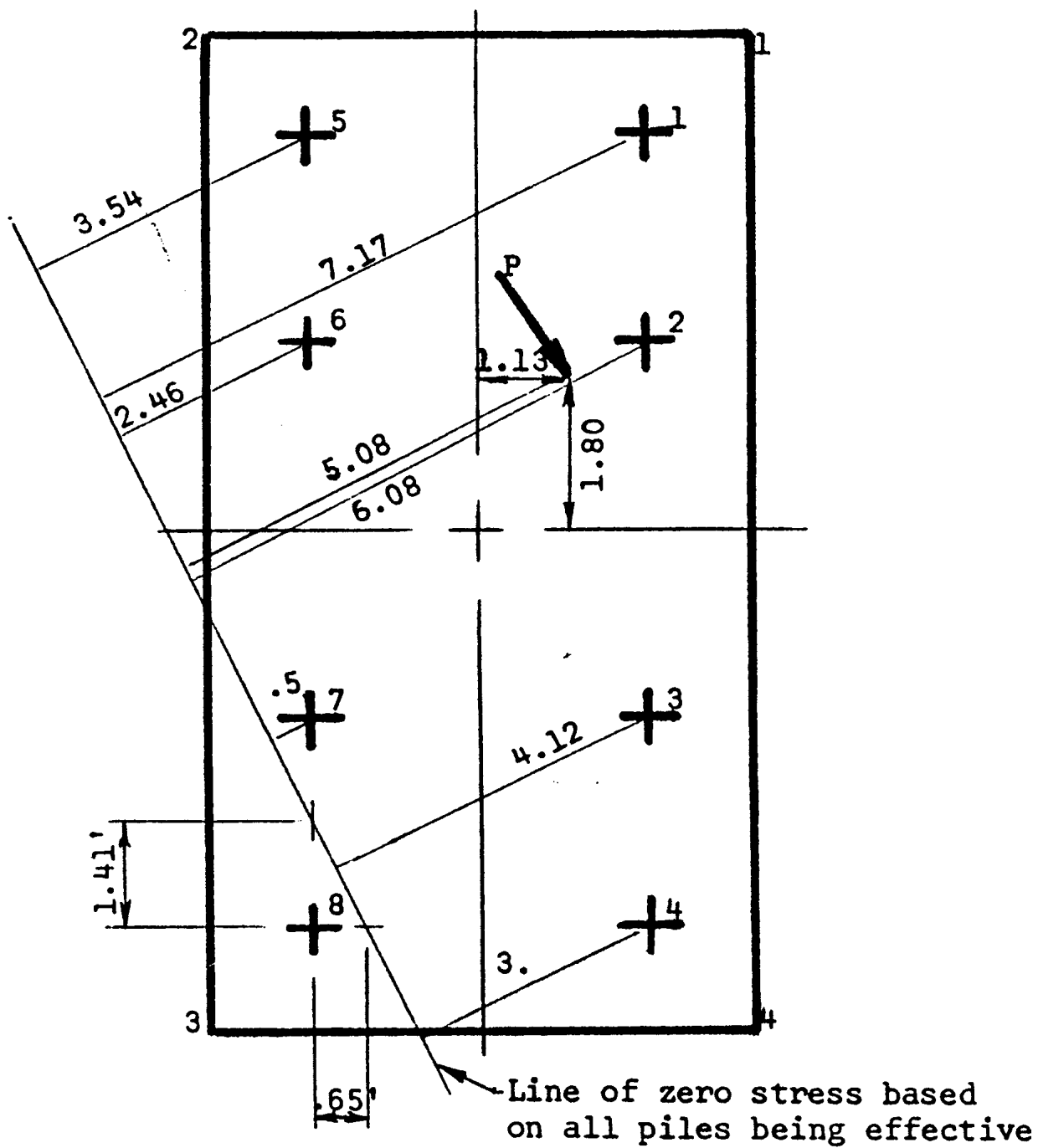
$$\text{Distance from Pile 8: } Y = \frac{10.2}{68.8} \times 9.5 = 1.41 \text{ feet}$$

$$X = \frac{10.2}{62.6} \times 4.0 = .65 \text{ feet}$$

Determine eccentricities of axial load:

$$e_x = \frac{500}{444} = 1.13 \text{ feet}$$

$$e_y = \frac{800}{444} = 1.80 \text{ feet}$$



Compute new section properties and pile loads:

About line of zero stress,  $I = \sum d^2$

$$I = 3.54^2 + 7.17^2 + 2.46^2 + 6.08^2 + 5^2 + 4.12^2 + 3^2 = 133.21 ft.^4$$

$$S = \frac{I}{d} = 133.21 / 7.17 = 18.60 ft.^3$$

$$\text{Moment} = 444 \times 5.08 = 2255. \text{ Ft. Kips}^2$$

$$* F1 = M/S = 2255/18.6 = 121 \text{ Kips}$$

$$F2 = 121 \times 6.08/7.17 = 102.5$$

$$F3 = 121 \times 4.12/7.17 = 69.5$$

$$F4 = 121 \times 3/7.17 = 50.6$$

$$F5 = 121 \times 3.54/7.17 = 59.6$$

$$F6 = 121 \times 2.46/7.17 = 41.5$$

$$F7 = 121 \times 5/7.17 = 8.4$$

$$\text{Summation of pile loads} = 453 \text{ Kips}$$

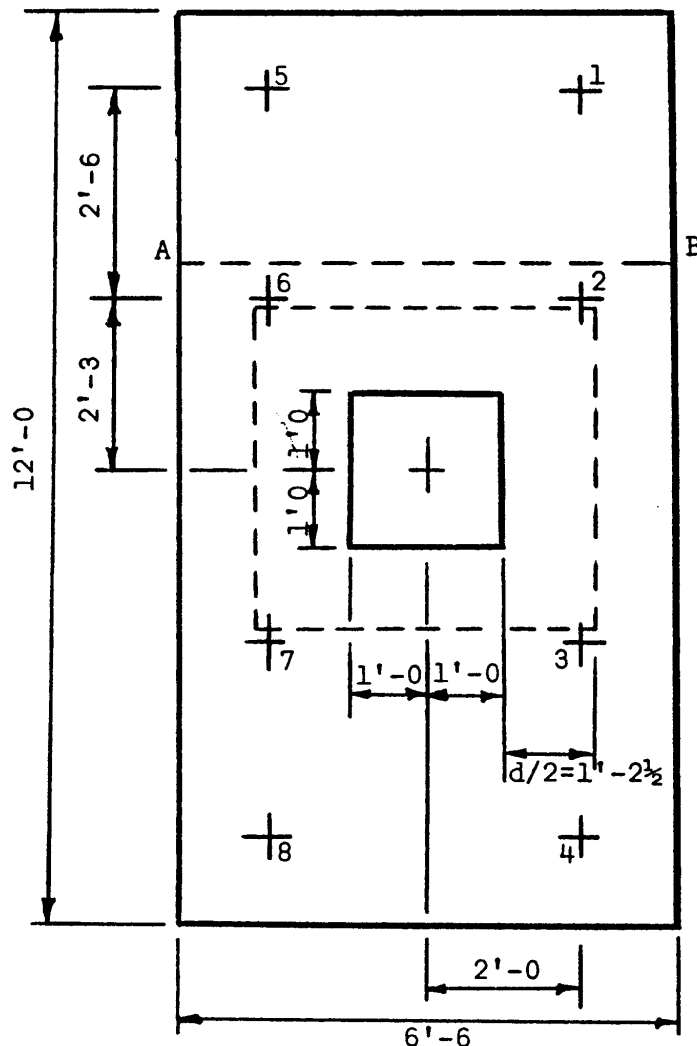
Since 453 is greater than 444, the actual position of the line of zero stress is closer to the point of maximum stress. However, the pile forces obtained are well within the acceptable limits of accuracy.

The distance used in this method of analysis are scaled. The answer obtained is identical to the one obtained using the section properties of all piles. It is not necessary to perform an exact analysis when a small percent of the area between the outside piles is in uplift.

\* Allowable pile stress for Group 2 Loading:

$$110 \text{ Kips} \times 1.25 = 137.5 \text{ Kips}$$

## B. Shear Strength



Because piles 1, 2, 3 and 4 fall inside a line at distance  $d$  from the face of the column a check of shear strength in the transverse direction is not required.

For shear in the longitudinal direction one-way action is checked.

Shear at AB = pile forces minus footing and soil loads.

For 3' footing,  $d = 36 - 7 = 29"$

$$V_u = 1.3(F1 + F5) - (6.5 \times 2.58 \times 3 \times .15)1.3 - (6.5 \times 2.58 \times 1 \times .12)1.3$$

$$V_u = 235 - 9.8 - 2.6 = 222.6$$

Actual Shear Stress:

$$v_u = V_u / \phi b_w d$$

$$v_u = 222.6 / (.85 \times 78 \times 29) = .116 \text{ ksi}$$

$$\text{Allowable Shear Stress} = 2\sqrt{f_c'} = 2\sqrt{3500} = 118 \text{ psi}$$

Two way action or peripheral shear:

$$P = 1.3(400) = 520 = V_u$$

$$v_u = V_u / \phi b_o d = 520 / (.85 \times 4 \times 4.42 \times 12 \times 29)$$

$$v_u = .099 \text{ ksi}$$

## C. Design Longitudinal Steel

Moment at face of column

$$M_u = [(F1 + F5)x3.75 + (F2 + F6)x1.25]1.3 \\ - (5x6.5)(3.08x.15 + 1x.12)x2.5)1.3 = 1055.6 \text{ ft.kips}$$

Determine steel area required:

$$R_u = \frac{M_u}{\phi b d^2} = \frac{1055.6}{.9(6.5)x29^2} = .214$$

From tables  $p = .0037$

$$\text{Area of steel} = .0037x78x29 = 8.37 \text{ in.}^2$$

Use 11 #8 bars.

#### D. Design Transverse Steel

Because the piles are less than 2/3 of the effective footing depth away from the face of the column, bending is not considered in the transverse direction. Place a nominal amount of reinforcing steel in the footing, like #5 at 12".

#### E. Pile Group Action

This example assumes that a reduction for Pile Group Action must be computed for a full friction pile. Refer to Chapter 11 of this manual, or the AASHTO Specifications for a definition of the Converse - LaBarre equation.

$$E = 1 - (\phi(n-1)m + (m-1) / 90mm(AASHTO1.4.4(G))) \\ n = 4, m = 2, d = 1, s = 4$$

$$E = 1 - [14(3)(2) + 1(4)] / (90x4x2) = .88$$

$$\text{Allowable Pile Load} = 110 \times 1.25 \times .88 = 121 \text{ kips (Drive pile to 110 kips)}$$

#### F. Footing Dimensions

Footing size - 12' x 6'-6 x 3'

Bar steel - Longitudinal - Use 11-#8 bars

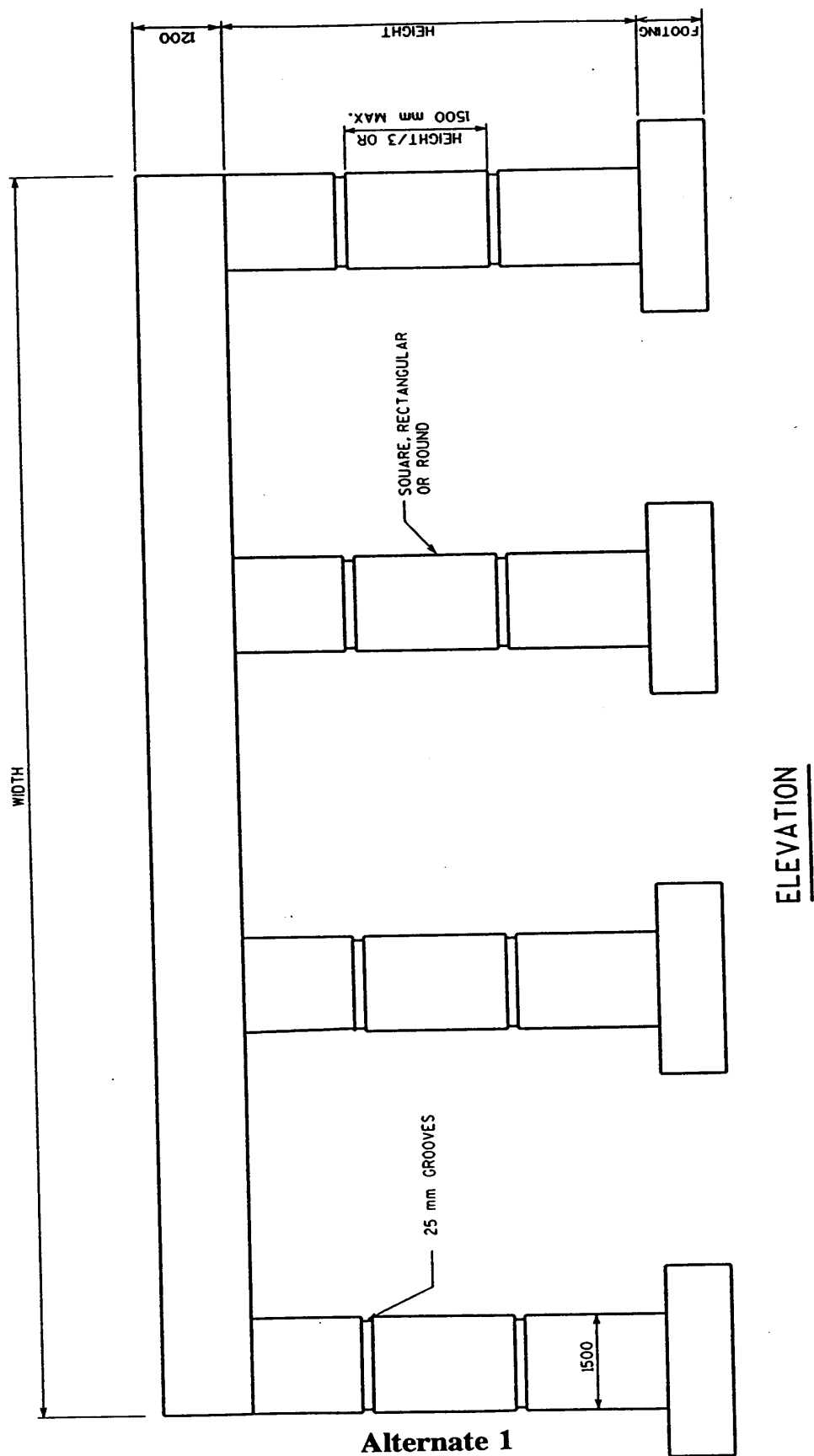
Transverse - Use #5 bars at 12 inches

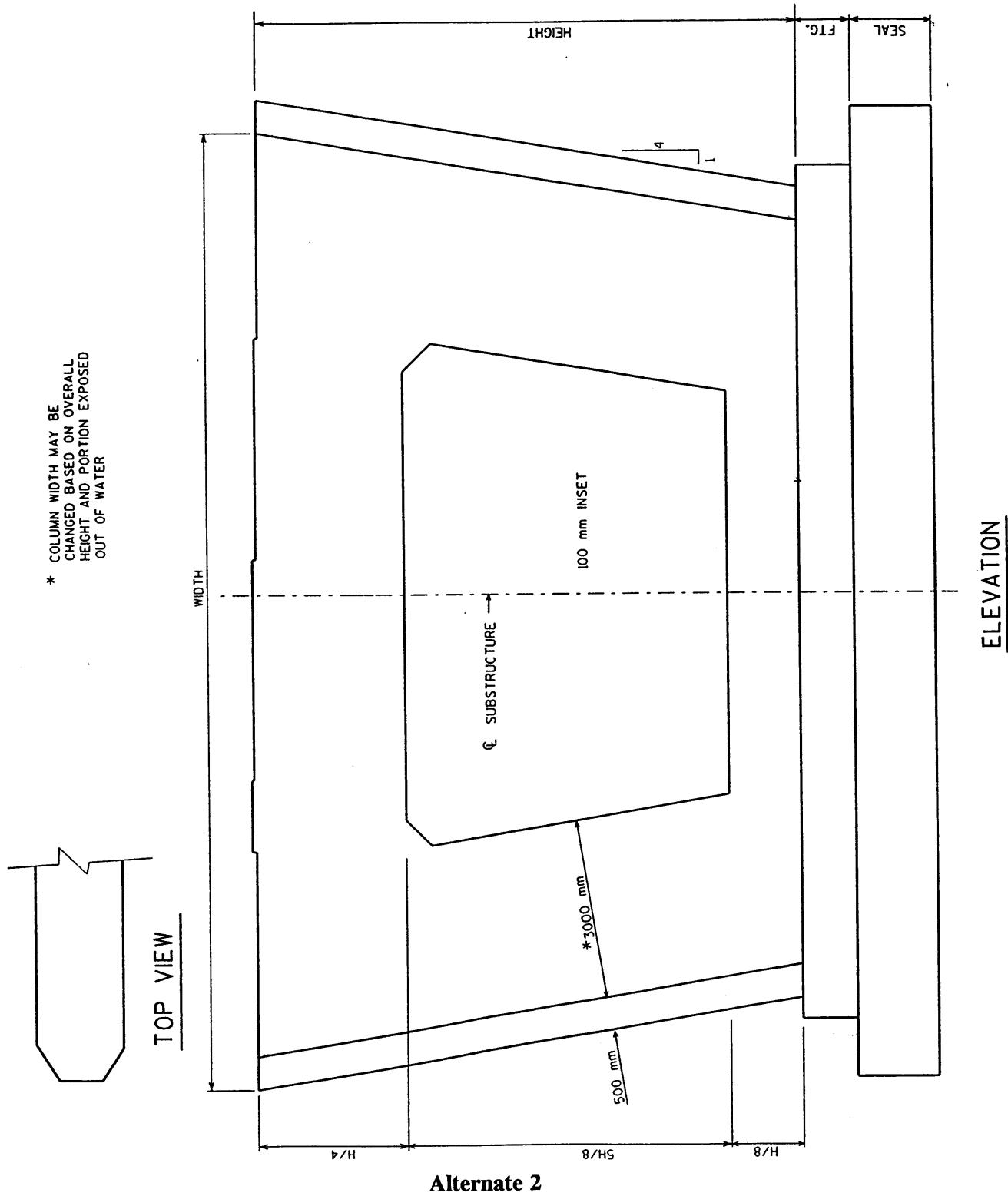
**13.7 QUANTITIES**

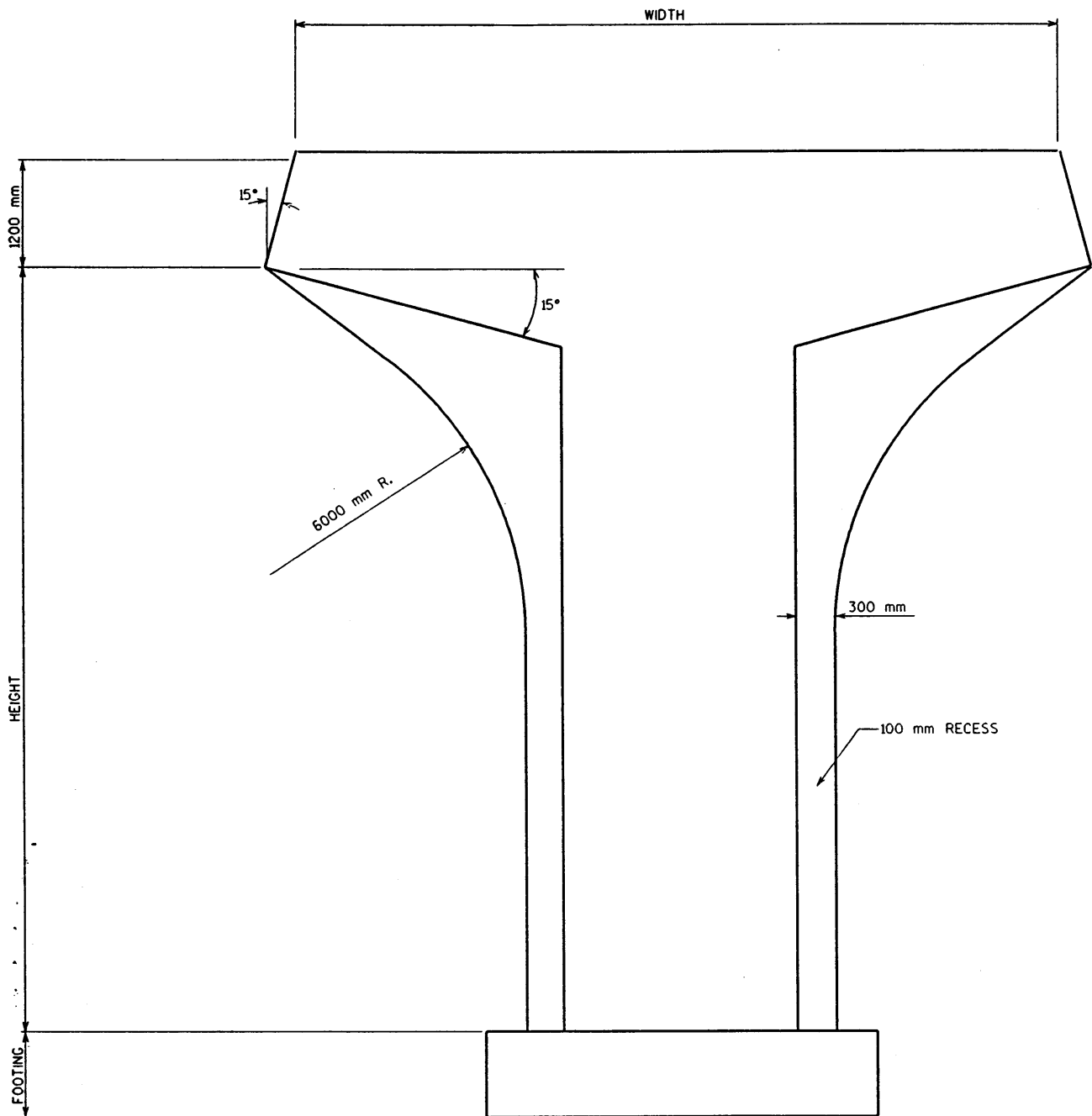
Consider the "Upper Limits for Excavation" for piers at such a time when the quantity is a minimum. This is either the existing ground line or the finished graded section. Indicate in the general notes which value is used.

Granular backfill is not used at piers except for special conditions.

Compute the concrete quantities for the footings, columns, and cap and show them for each one on the final plans.





ELEVATION

Alternate 3

### PILE ENCASED PIER CONSTRUCTION

The following procedures shall be employed in the construction of pile encased piers which are constructed without cofferdams. The contractor shall have the option of driving piling prior to excavation. Excavation shall proceed in the following manner.

The area occupied by the pier and form work shall be over-excavated a minimum of 2 feet (600 mm) below the bottom of pier elevation. This area shall be partially filled with clean stone, to a point approximately 6 inches (150 mm) below concrete grade. The preassembled form shall then be lowered into position. The form shall then be properly positioned, plumbed, and secured to prevent movement during the pouring operation. Clean stone shall then be placed inside the form to insure that concrete cannot leak out from under the form, as well as to attain the bottom of pier elevation.

Reinforcing steel may be incorporated in the form prior to setting, or it may be placed after the form has been set. A sufficient number of bar chairs shall be used to insure that proper clearances are achieved.

Concrete shall then be placed through the use of a tremie, or tremies - dependent upon the width and height of the pier. The movement of tremies and recharging shall be limited as much as possible. Placement operations shall be in accordance with Subsection 502.3.6.3 of the Standard Specifications. When the concrete reaches an elevation which is above the water elevation, the surface shall be checked for any unsatisfactory materials which may have been forced upward as the concrete was being deposited. Any deleterious materials found shall be removed at that time from inside the form. Further concrete placement may then proceed, using standard placement methods. Silt curtains or screens, as specified on the plans, shall be employed to limit downstream siltation. Displaced water shall be treated by filtration, settling basin or other means sufficient to reduce the cement content before being discharged into the stream.

After the formwork has been removed, the excavated area shall be backfilled to the original stream bed elevation.